Annual Project Summary Report for 1/01/2005 to 12/31/2005

Seismic Compression of Compacted Fill Soils

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NON-TECHNICAL PROJECT SUMMARY

During earthquakes ground deformations can occur in unsaturated soils that cannot be explained by well-known phenomena such as soil liquefaction or landsliding. Such phenomena, referred to as seismic compression, are relatively poorly understood. This project involves advanced cyclic soil testing in the laboratory to identify the fundamental factors that control ground deformations during seismic compression. Previous research on seismic compression has focused principally on clean sandy soils. The present study is extending this work to soils with high fines contents and plastic fines. The results will be useful for assessing the vulnerability of sites with unsaturated soils to strong earthquake shaking.

INVESTIGATIONS UNDERTAKEN

This project is investigating the seismic compression potential of soils with variable plasticity. Preliminary work revealed a significant effect of time (ageing) effects on seismic compression potential. Much of the work to date has been directed towards better understanding these time effects. Preliminary results are summarized below and by Duku et al. (2006a). Additional work has involved a thorough characterization of the control capabilities of the digitally-controlled simple shear (DC-SS) testing device at UCLA.

RESULTS

Ageing Effects on Seismic Compression

When a stress increase is imposed upon a sample of unsaturated soil, a relatively rapid period of compression occurs, followed by an extended period of volumetric creep (secondary compression). The initial compression is similar to primary consolidation of saturated soils, but occurs much more quickly.

Ageing effects are derived from the time allowed for secondary compression following the initial (primary) consolidation. Representative results are shown here for two soils; a non-plastic silty sand and a relatively plastic clay. The non-plastic soil has fines content (FC) = 31%, liquid limit (LL) = 28, and plastic limit (PL) = 0. The plastic soil has FC = 74%, LL = 64, PL = 31, and PI = 33. Both the non-plastic and plastic soils were wetted to a saturations (S = 60%), but compacted to a relative compactions (RC) of 88 and 80 %, respectively. The specimens were then subjected to compression under a sustained static load of 27.6 and 48.3 kPa, respectively, for 2 minutes and 2 hours, as shown in Figure 1(a) for the silty sand and Figure 2(a) for the clay.

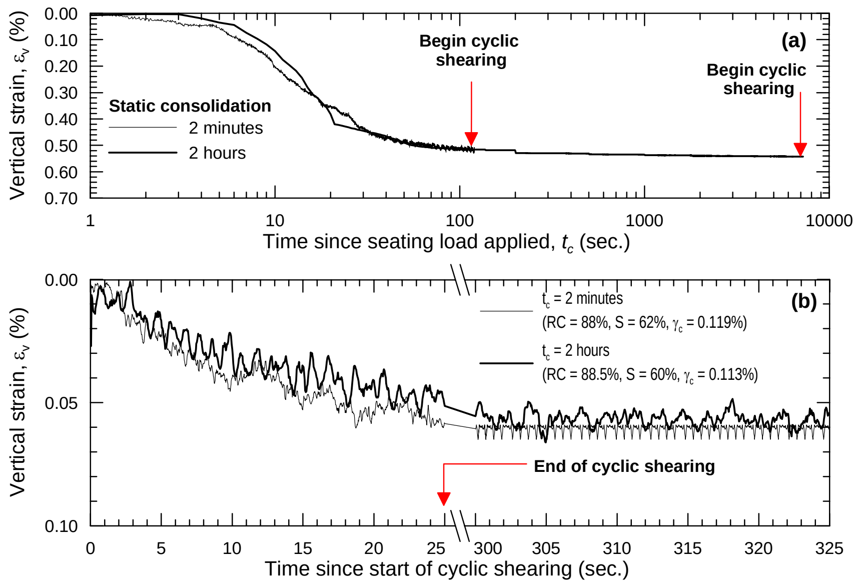

Figure 1. (a) Compression histories of non-plastic soil and (b) compression during and following cyclic shearing

At the end of consolidation, the specimens were sheared at a constant cyclic shear strain amplitude γ_C = 0.1 % applied at a 1 Hz loading frequency for 25 cycles. Vertical strain was measured during the cyclic shearing, with the results shown in Figure 1(b) and 2(b).

For the non-plastic soil (Figure 1), the cyclic vertical strain accumulation is practically the same for the two specimens, indicating no effect of consolidation time. On the other hand, the seismic compression of the plastic soil is significantly different for the specimens subjected to short and long consolidation times. The role of these time (or ageing) effects is to decrease the seismic compression potential, as shown in Figure 2(b).

Another effect evaluated during this testing program is post-shaking soil compression. No such compression is observed for the non-plastic silty sand, as can be seen by the flat ε_V-time plots in Figure 1(b). On the other hand, for the clayey soil, the specimen consolidated for less time experienced more post-shaking compression, although in both cases this post-shearing compression is negligible relative to that induced by cyclic loading.

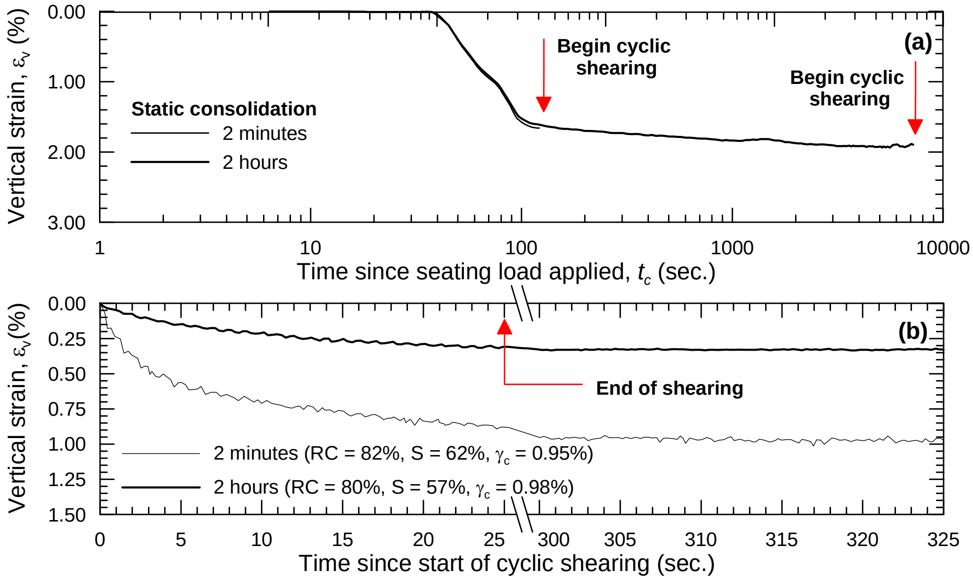

Figure 2. (a) Compression histories of plastic soil under sustained static load and (b) compression during and following cyclic shearing.

<u>Comparison of Results for Clays and Sands</u>

Vertical strains from seismic compression for soils with significant fines content (tested in this research) are compared with seismic compression of clean sands (tested in previous research, Stewart et al., 2004) at comparable densities in Figure 3. Figure 3(a) presents results for the non-plastic silty sand, for which vertical strains are seen to be within the range of the sand results, although slightly lower than the median. Figure 3(b) presents results for the clayey soil, for which vertical strains are seen to be significantly lower than those for clean sand. The significant effect of consolidation time is also apparent from Figure 3(b). It is noteworthy, however, that even at large consolidation times, the compacted clay has non-zero seismic compression, indicating that such effects would need to be considered in seismic design in areas that might be subject to very strong shaking.

<u>Control Characteristics of Test Device</u>

An extensive study has been conducted to document the control characteristics of the UCLA DC-SS device being used in this project. This intent of this work is to improve the device and to gain a fundamental understanding of its capabilities. Knowledge of these capabilities guides the development of meaningful test protocols for engineering research. The major improvement to the device during the reporting period involved upgrading the controller from a Proportional-Integral-Derivative (PID) controller to a multiple-input multiple-output (MIMO) controller.

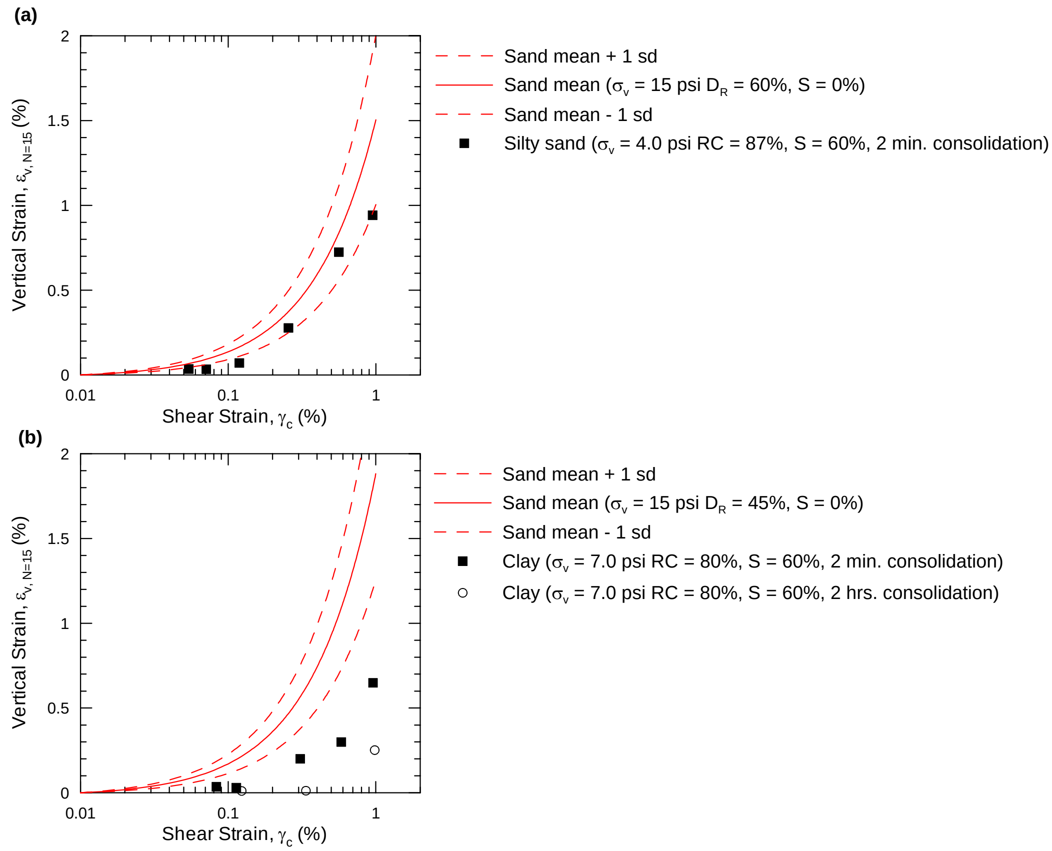

Figure 3. (a) Seismic compression comparison of silty sand and sand at comparable density and (b) seismic compression comparison of clay at different consolidation times and sand at comparable density.

The device performance is monitored by a normalized root mean squared tracking error (ε_{RMS}), which is computed from a command (x^c) and feedback (x^f) signal as follows:

$$\text{Normalized RMS tracking error, } \varepsilon_{RMS} = \frac{\sum_{i=1}^{N}\left(x_i^c - x_i^f\right)^2}{\sum_{i=1}^{N}\left(x_i^c\right)^2} \qquad (1)$$

where N is the number of data points in the command and feedback signals.

Figure 4(a) shows a comparison of tracking errors with the device operated in PID and MIMO control mode. The input signal is a sinusoidal wave of f = 1 Hz at various amplitudes. Figure 4(b) shows tracking errors as a function of frequency at amplitudes of 0.02032 mm and 0.2032 mm.

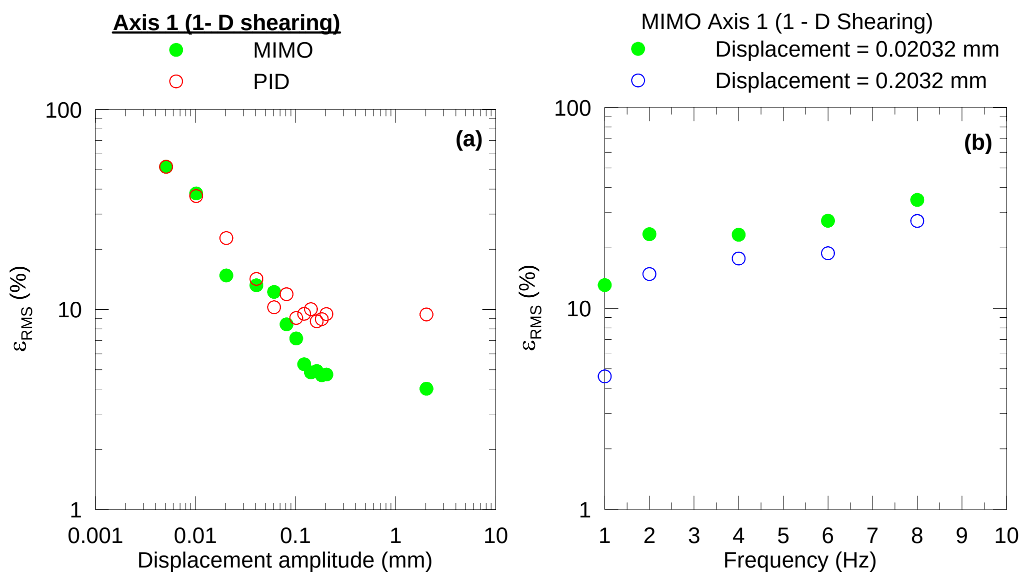

Figure 4. Variation of normalized root mean square of tracking errors for uni-directional shaking with (a) displacement amplitude and (b) loading frequency.

The results in Figure 4(a) show that errors increase as amplitude decreases. At displacements < 0.01 mm, with errors exceeding about 20% of the signal, it can be said that the device has difficulty achieving good control. Note that a displacement amplitude of 0.01 mm corresponds to a shear strain of 0.05% for a 2 cm tall specimen (typical height). Those control difficulties arise because of noise introduced by the hydraulic control system, which produces a feedback signal with a mean amplitude of approximately 0.0053 mm. Figure 1(a) also shows that at shear strains larger than about 0.08 mm, tracking errors are smaller with the MIMO controller than with PID.

Figure 4(b) shows that ε_{RMS} increases with frequency, this reduction of control with frequency occurs when the command frequencies become closer to the oil column frequency (f_{oil}) of the actuator, which is approximately 16 Hz.

One of the principal attributes of the UCLA DC-SS device is its ability to achieve highly accurate control of broadband signals simultaneously in two horizontal directions of shaking (bi-directional control). An example of a bi-directional, broadband test is shown in Figure 5. The ground motion record used in Figure 5 is from the 1999 Chi-Chi Taiwan mainshock (TCU 065 west component). That record was used in both uni-directional and bi-directional tests. The record was scaled to a peak amplitude of 0.2032 mm, corresponding to a peak shear strain of approximately 1% for specimen height of about 2 cm. The response of three soil specimens is shown. Two were tested in uni-directional mode, the third in bi-directional mode. The sum of the strains generated from the uni-directional tests is seen to be generally similar to that for the bi-directional test.

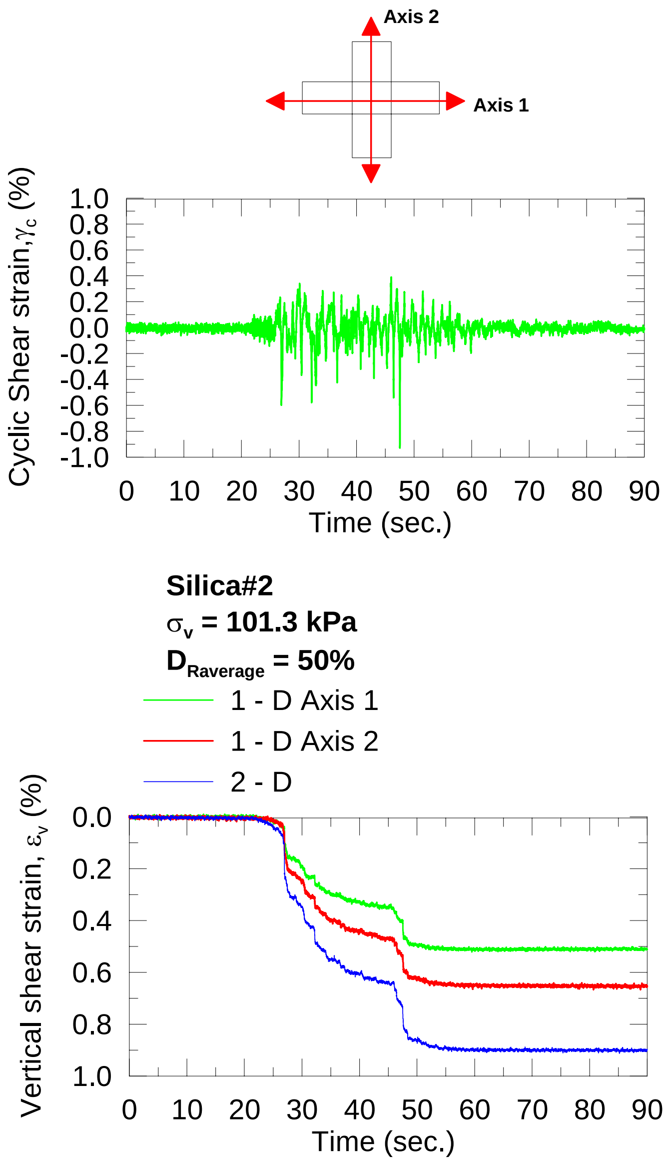

Figure 5. Comparison of vertical strains of clean sand specimens prepared to comparable relative densities and confined at 101.3 kPa to uni-directional and bi-directional loading

REPORTS PUBLISHED

Further information on the role of time effects on seismic compression of plastic soils and the UCLA DC-SS device can be found in Duku et. al. (2006a) and Duku et. al. (2006b), respectively.

AVAILABILITY OF DATA

Data is available from the project web site at http://www.nees.ucla.edu/fills/index.htm

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